Homer Hadley (Interstate 90) Floating Bridge Test Program for Light Rail Transit

Test Report

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1. EXECUTIVE SUMMARY

In September 2001, KPFF prepared a report titled "Homer Hadley (Interstate 90) Floating Bridge – Draft Structural Feasibility Study Light Rail Conversion". In the report, analytical studies were performed on the floating pontoons and elevated superstructure to determine if the existing bridge structure could support Sound Transit's current LRT loads. Pontoon freeboard loss, rotation, moment, and torsion were obtained and used to evaluate the response of the bridge to the static loads applied at mid-span and at the west end. The results of the study indicated that for LRT live load alone, freeboard loss on the bridge is significant, structure bending-moments appear to be within acceptable levels, torsion stresses are high, and structural retrofit measures are required for the elevated steel superstructures at the east and west ends of the bridge. For rail system dead load added to the bridge, weight mitigation measures were proposed to mitigate freeboard loss. The report also recommended that more global analysis of the bridge be performed to verify these results.

The objective of this report is to perform full-scale load tests to simulate the analytical studies of bridge response to Sound Transit LRT live load included in KPFF's previous report. A test program was selected over a more refined computational analysis because it would eliminate the hydrodynamic modeling and geometric non-linear uncertainties typically associated with floating bridge dynamic analysis.

On September 16, 17, and 18, 2005, KPFF Consulting Engineers performed the full-scale load test of the I-90 Homer Hadley Floating Bridge, simulating light rail transit (LRT) live loading. The test involved eight flatbed trucks that were loaded to approximate the weight of light rail vehicles (148,000 pounds each, with two four-truck combinations each simulating a four-car light rail train). Sensitive instrumentation installed on the bridge captured data as the bridge responded to the movement of the trucks. Both static and dynamic tests were performed.
The load test provided real-time information about how the floating bridge would respond to the weight of light rail trains. Test results indicate that:

1. Bridge response correlates well with response predicted using previously developed analytical methods.

2. There appears to be no definitive trend when comparing global bridge response due to static loading to that for dynamic loading. Globally, the bridge responded similarly for static and dynamic loading.

3. The continued use of previously developed analytical methods used to study the response of the bridge to LRT live loading is acceptable.

Additional analysis was performed to assess the global capacity of the bridge at mid-span and near the west expansion joint to support LRT live load in combination with other design loads. The analysis was performed following WSDOT's design criteria document used for design of the bridge. Results indicate that WSDOT's strength and serviceability criteria for pontoon global response are met at mid-span and at the far west end of the bridge for live loading due to two (2) tracks of Sound Transit's LRT system in combination with HS25 traffic on the westbound roadway.

The load test and analysis confirm that the Homer Hadley (Interstate 90) Floating Bridge can structurally carry the operational loading from the proposed light rail trains, the initial computer

model can be used with the appropriate modification factors identified in the test program, and the bridge can be retrofitted to handle the dynamic load of the proposed light rail system. An important next step in the analysis of a light rail system along Interstate 90 across Lake Washington is a preliminary study of the design of the track system to accommodate motions at the floating bridge expansion joints.

2. INTRODUCTION

In September 2001, KPFF prepared a report titled "Homer Hadley (Interstate 90) Floating Bridge – Draft Structural Feasibility Study Light Rail Conversion". In the report, analytical studies were performed on the floating pontoons and elevated superstructure to determine if the existing bridge structure could support Sound Transit's current LRT loads. The analysis of the bridge incorporated a two-dimensional continuous stiffness computer model of the floating structure supported on an elastic foundation and subject to static LRT live loading located in the existing HOV lanes. In the computer model, LRT live loads were placed at mid-span and near the west transition span expansion joint in arrangements that simulated trains bypassing and about-to-bypass, with one train located on each track. Bridge pontoon freeboard loss, rotation, moment, and torsion were obtained, and used to evaluate the response of the bridge to the applied static loads. The results of the study indicated that for LRT live load alone, freeboard loss on the bridge is significant, structure bending-moments appear to be within acceptable levels, and torsion stresses are high. The report also recommended that more global analysis of the bridge be performed to verify these results.

The objective of this report is to perform full-scale load tests to simulate the analytical studies of bridge response to Sound Transit LRT live load included in KPFF's previous report. A test program was selected over a more refined computational analysis because it would eliminate the hydrodynamic modeling and geometric non-linear uncertainties typically associated with floating bridge dynamic analysis.

On September 16, 17, and 18, 2005, KPFF Consulting Engineers performed the full-scale load test of the I-90 Homer Hadley Floating Bridge, simulating light rail transit (LRT) live loading. The intent of this report is to:

- Present test program description and purpose.
- Present the results of the test program.
- Present comparisons of test results to bridge response predictions made by analytical methods.
- Present conclusions on the applicability of previously used analytical methods.
- Incorporate predicted bridge response to LRT loading into the original design criteria used by WSDOT to design the floating bridge.
- Conclude on the structural effects of LRT on the floating bridge.

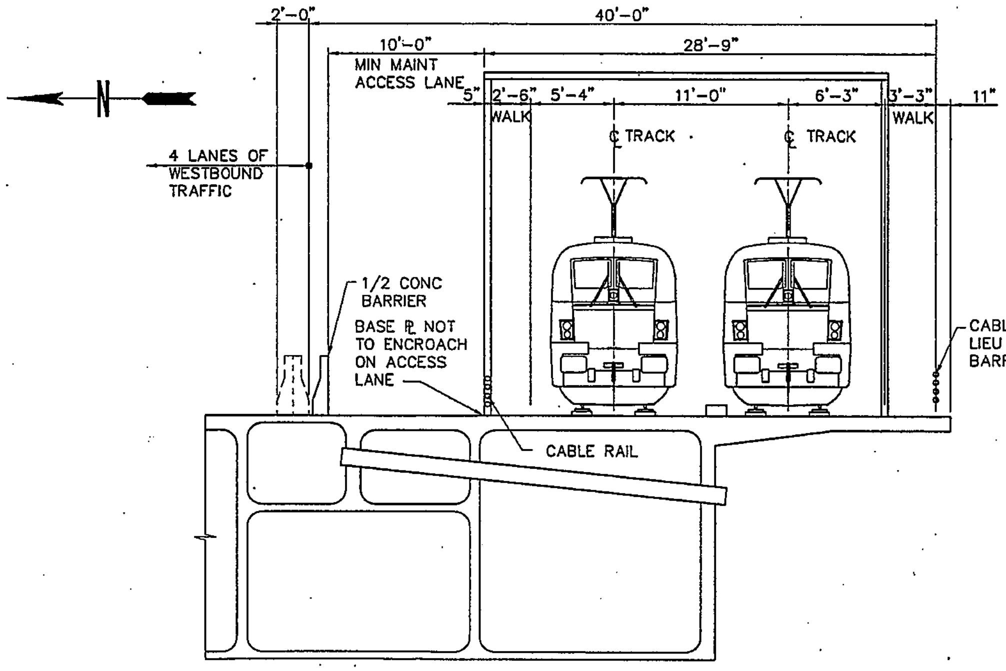
3. TEST PROGRAM OVERVIEW

The test program involved performing full-scale load tests on the floating bridge and comparing measured bridge response to that predicted by the analytical studies. Global bridge response under both static and dynamic load conditions was measured. Response parameters measured include:

- Freeboard loss

- Bridge rotation
- Cantilever tip deflections along the south edge
- Horizontal and vertical deflections at the expansion joint
- Vertical and horizontal accelerations
- Global pontoon strain due to combined moment, torsion, and shear

Static load conditions were simulated by placing fully-loaded test vehicles at specific locations within the existing HOV lanes of the bridge at mid-span and near the west transition span expansion joint. Test vehicle location within the existing HOV lanes corresponded to track location 3 identified in the previous analytical study, which accommodates the addition of a fourth westbound traffic lane.



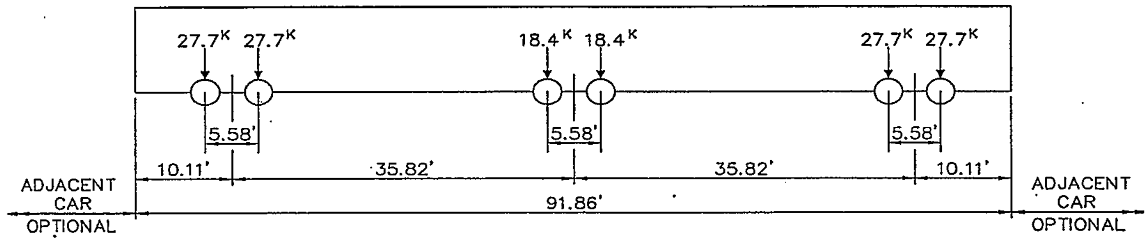
<u>Alternative LR (MOD) - 3</u>

Dynamic load conditions were simulated by driving fully-loaded test vehicles in train formation at 30-miles-per-hour within the existing HOV lanes of the bridge. Tests were conducted for single trains traveling in both directions along the entire length of the bridge and for trains bypassing at mid-span and near the west transition span expansion joint.

Response data for the static and dynamic test was collected in real-time as the test vehicles moved along the bridge. Baseline readings for the empty bridge condition and for the condition with traffic in the westbound lanes were also taken.

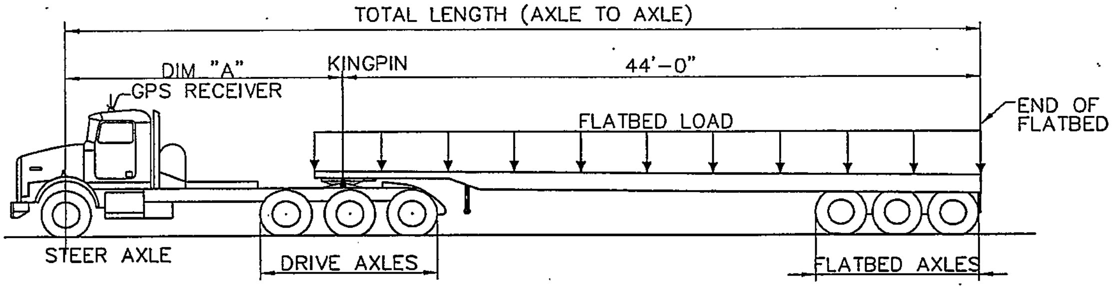
a. Simulated LRT Vehicle

The test program simulated the uniform LRT live load applied to the bridge for a 4-car train. One car of the simulated Sound Transit LRT train is shown below:



LRT VEHICLE LIVE LOADING

The uniform load applied to the bridge for the vehicle shown above is approximately 1600 pounds per lineal foot (plf). This live load is applied over a distance of 370-feet for a 4-car train. The test vehicles used to simulate the uniform LRT live load consisted of flatbed trucks loaded with weights to approximate the total gross weight of the LRT vehicles. Axle spacing and axle load for the loaded flatbeds trucks did not match those of the simulated LRT vehicle. Shaughnessy and Co. provided, loaded, and operated the test vehicles. The general arrangement of the typical test vehicle is shown below:



TYPICAL TEST VEHICLE
NOT TO SCALE

A "train" of test vehicles consisted of four (4) trucks spaced evenly apart to approximate the total 370-foot length of the simulated 4-car train. The simulated trains were placed on the bridge at mid-span and near the west transition span expansion joint to replicate the loading applied in the analytical study. The trains were arranged to simulate the bypass and about-to-bypass condition, with one train located in each HOV lane. Train "A" was located in the north HOV lane and Train "B" was located in the south HOV lane. Load added to the flatbed of each truck consisted primarily of steel and concrete material. The target gross vehicle weight (GVW) for each truck was 148,000-lbs to approximate the weight of one LRT car. The tables below summarizes the actual weight of each test vehicle as documented by Shaughnessy and Co.

Train A - Located in North HOV Lane

Vehicle	Total Documented GVW, lbs	% of Target GVW
A-1	147,080	99.3
A-2	147,700	99.8
A-3	146,340	98.9
A-4	144,217	97.4
Total Train Weight	**585,337**	**98.9**

Train B – Located in South HOV Lane

Vehicle	Total Documented GVW, lbs	% of Target GVW
B-1	148,370	100.3
B-2	145,430	98.3
B-3	147,511	99.7
B-4	147,510	99.7
Total Train Weight	**588,821**	**99.5**

b. Bridge Instrumentation and Response Measurement

Bridge response to the simulated LRT live loading was measured by instrumentation installed at five (5) stations located on the west half of the bridge. Since the floating pontoons are essentially symmetrical about the centerline of the bridge, global response of the entire bridge was evaluated by instrumenting only one-half of the structure. Factors influencing the selection of instrumentation locations included bridge response predicted in analytical studies, bridge accessibility and interior/exterior obstructions. Bridge instrumentation consisted of the following:

Sensor Type	Response Measured
Strain gages	Strains due to longitudinal moment & torsion
Tilt meters	Bridge rotation and cantilever tip deflection
Accelerometers	Vertical and horizontal acceleration
String Potentiometer	Expansion joint movement and freeboard loss
GPS receivers	Vertical/horizontal displacement and test vehicle location

Construction Technologies Laboratories (CTL) furnished, installed and operated data collection for the strain gages, tilt meters, accelerometers and string potentiometers. WSDOT's Strategic Planning & Programming, Geographic Services group furnished, installed, and operated data collected for the GPS receivers. Sensor readings were collected continuously throughout the nights of the test using computerized data collection programs on portable computers. All data collection systems were time synchronized to a common time reference and imported into Excel spreadsheets. Readings were taken under the following conditions:

1. No vehicles on bridge (baseline condition).
2. Traffic on westbound lanes of bridge, with HOV lanes closed.

3. Static load tests of simulated LRT vehicle, with no other traffic on the bridge.
4. Dynamic load tests of simulated LRT vehicle, with no other traffic on the bridge.

Figures 1 through 8 show where and how instrumentation was installed on the bridge. Additional information about the instrumentation systems is located in Appendix B.

c. Static Load Tests

Static load tests consisted of parking fully-loaded test vehicles in various arrangements within the existing HOV lanes at mid-span and near the west transition span expansion joint to replicate the loading applied in the analytical study. The test vehicles were positioned to correspond to track location 3. The following is a summary of the static load tests performed:

Case	Description
S1	Train A at mid-span in north HOV lane
S2A	Trains A & B bypassing near mid-span
S2B	Trains A & B bypassing 510 feet west of mid-span
S3	Trains A & B about to bypass near mid-span
S4	Train B at mid-span in south HOV lane
S5	Train A near west transition span expansion joint in north HOV lane
S6	Trains A & B about to bypass near west transition span expansion joint
S7	Trains A & B bypassing near west transition span expansion joint
S8	Train B near west transition span expansion joint in south HOV lane

Test vehicle location for cases S2A and S2B was selected to maximize response measured at instrumentation stations 1 and 2. Instrumentation data was collected for approximately 3-minutes for each test case after the test vehicles were in position. For test case S1, the four (4) test vehicles within Train "A" were positioned on the bridge one at a time. Data was collected for approximately 1-minute after each test vehicle was in position. Figure 9 shows test vehicle location for each test case.

Data was extracted for each of the above test cases based on the documented times they occurred. The following chart summarizes the time range for each static test case:

Date	Case	Time Range
9/16/2005	Truck A1 in position	23:14:23 – 23:15:25
	Truck A2 in position	23:20:52 – 23:22:01
	Truck A3 in position	23:26:26 – 23:27:38
	S1	23:36:05 – 23:39:13
	S2A	23:57:01 – 00:00:11
9/17/2005	S2B	00:29:51 – 00:33:18
	S3	00:45:16 – 00:48:19
	S4	00:56:13 – 00:59:17
	S5	01:50:52 – 01:53:56
	S6	02:07:27 – 02:10:28
	S7	02:18:36 – 02:21:47
	S8	02:25:54 – 02:28:58

d. Dynamic Load Tests

Dynamic load tests consisted of driving fully-loaded test vehicle trains along the entire length of the bridge at 30-miles-per-hour. 30-miles-per-hour was selected based on the comfort level of the test vehicle drivers. The drivers were directed to drive along the centerline of the existing HOV lanes. The following is a summary of the dynamic load tests performed:

Case	Description
D1	Train B traveling eastbound in south HOV lane
D2	Train A traveling eastbound in north HOV lane
D3	Train B traveling westbound in south HOV lane
D4	Train B traveling eastbound in south HOV lane, bypassing Train A parked in north HOV lane at mid-span
D5	Train B traveling westbound in south HOV lane, bypassing Train A parked in north HOV lane near west transition span expansion joint
D6	Train B traveling eastbound in south HOV lane, bypassing Train A parked in north HOV lane near west transition span expansion joint
D7	Trains A & B bypassing near mid-span (Train A heading eastbound, Train B heading westbound)
D8	Trains A & B bypassing near west transition span expansion joint (Train A heading eastbound, Train B heading westbound)
D9	Trains A & B bypassing approximately 510 feet west of mid-span (Train A heading westbound, Train B heading eastbound)
D10	Trains A & B bypassing near west transition span expansion joint (Train A heading eastbound, Train B heading westbound)
D11	Trains A & B bypassing near west transition span expansion joint (Train A heading westbound, Train B heading eastbound)

Dynamic tests D8, D10, and D11 were performed such that the trains would bypass as close as practical to the west transition span expansion joint on the pontoon side of the joint. The trains bypassed each other at different locations with respect to the expansion joint for each test case. The trains bypassed closest to the expansion joint during test D11. GPS receivers were mounted to the lead truck of Trains "A" & "B" to record the position of the trains. Figure 10 shows test vehicle location for each test.

Data was extracted for each of the above test cases based on the documented times they occurred. The following chart summarizes the time range for each dynamic test case:

Date	Case	Time Range
9/17/2005	D1	03:41:25 – 03:47:35
	D2	03:54:01 – 04:00:00
	D3	04:16:36 – 04:22:32
	D4	04:53:15 – 04:59:20
	D5	05:30:33 – 05:37:45
9/18/2005	D6	01:13:20 – 01:18:47
	D7	00:21:28 – 00:28:21
	D8	01:54:26 – 02:01:20
	D9	02:37:00 – 02:44:00
	D10	03:13:17 – 03:20:56
	D11	03:55:00 – 04:02:46

e. Baseline Readings

Baseline readings for the empty bridge condition and with traffic on the westbound lanes were also taken. The times these readings were taken are summarized below:

Date	Case	Time Range
9/16/2005	Traffic in westbound lanes only	21:48:08 – 21:58:15
9/17/2005	Empty bridge	01:12:22 – 1:22:26

WSDOT provided traffic count data for the duration of the baseline test with traffic in the westbound lanes only. The total traffic count for the 10-minute interval of the baseline reading is approximately 359 vehicles. Complete traffic count data for September 16, 2005 is located in Appendix C.

4. TEST RESULTS AND OBSERVATIONS

a. Observed Site Conditions During Testing

Conditions during both nights of testing were similar. The following chart summarizes site conditions during the nights of testing.

Lake water level	20.4 feet (corps datum)
Average ambient air temperature	55 degrees F
Average wind speed	3 mph
Average pontoon anchor line tension	69 tons

Lake water level was obtained from the Army Corp of Engineers website. WSDOT provided records for weather conditions and pontoon anchor line tension. Appendix D includes complete records for the above.

b. Freeboard Loss Along South Side of Bridge

Two systems were used to measure freeboard loss along the south side of the bridge; GPS receivers mounted to the south traffic barrier and string potentiometers (POTs) that were part of a counterweight system mounted to the south traffic barrier. GPS receivers were installed at instrumentation stations 1, 2, 3, and 5. The string POT sensors were installed at stations 1, 2, and 5. Readings from both systems directly indicate the vertical displacement of the south edge of the bridge due to applied loading. Both systems operated properly during the tests and no discrepancies in the data were found.

Comparison of measured response from both data collection systems to predicted response for the static load tests is shown in Tables 1 & 2. The data is shown graphically in Figures 11 through 19. Predicted response of the bridge was obtained from the previously developed analytical model, with loads applied at track location 3. On average, freeboard loss reported by the GPS system is 0.5-inches greater than that reported by the string POT system. There is good correlation between measured and predicted freeboard loss. The graphs for the load tests with trains near mid-span show that measured deflection is less than predicted deflection immediately adjacent to the applied load, but greater than predicted away from the applied load. The measured distribution of deflection along the south edge of the bridge when loaded at mid-span appears to be more gradual than predicted. The more gradual distribution of bridge deflection may be due to the pontoon structure having a greater stiffness in flexural bending and torsion than estimated.

The graphs for load tests with trains near the west end of the bridge show that measured response is very close to predicted response near the expansion joint, but is typically greater than that predicted away from the applied load. Similar to the observed behavior at mid-span, it appears that the measured distribution of deflection along the south edge is more gradual than predicted when the bridge was loaded near the west end. The more gradual distribution of bridge response towards the west end of the bridge may be due to the pontoon structure having a greater stiffness in flexural bending and torsion than estimated.

Maximum freeboard loss for dynamic test cases D4 through D11 is summarized in Tables 3 & 4. The appropriate static test results are included in the table for comparison. The freeboard

loss values shown correspond to the maximum freeboard loss measured at the station immediately adjacent to the location where the trains bypass each other. The time stamp corresponding to the maximum freeboard loss reading at that station was used to extract data for the other stations. The data is shown graphically in Figures 12, 13, and 18.

Although there are differences in measured freeboard loss between the static and dynamic load tests, no definite trend is evident. On average, the order of magnitude of the difference in bridge response between static and dynamic loading is relatively small compared to the overall deflection of the bridge. There does not appear to be an amplification or dampening of bridge response due to dynamic loading.

c. Bridge Rotation

Tilt meters were installed inside the pontoons at instrumentation stations 1, 2, and 5 to measure global pontoon rotation in degrees. Data analyzed after the tests were complete indicated that the readings from the tilt meters at stations 2 and 5 included a significant amount of noise. The reliability of the data output by these sensors is questionable. As a backup, GPS receiver data was also used to compute pontoon rotation based on the difference in relative vertical displacement of the GPS units mounted to the north and south edges of the bridge. The GPS system operated properly during the tests and no discrepancies in the data were found.

Comparison of measured response from both data collection systems to predicted response for the static load tests is shown in Tables 5 & 6. The data is shown graphically in Figures 20 through 28. The same analytical model used to predict freeboard loss was used to predict pontoon rotation. There is good correlation between measured and predicted rotation. The graphs for the load tests with trains near mid-span show that measured rotation is less than predicted immediately adjacent to the applied load, but greater than predicted away from the applied load. Similar to the observation of freeboard loss, the measured distribution of pontoon rotation when loaded at mid-span appears to be more gradual than predicted. The more gradual distribution may be due to the pontoon structure having a greater torsional stiffness than estimated.

The graphs for load tests with trains near the west end of the bridge show that measured rotation is typically slightly less than predicted near the expansion joint, but is slightly greater than predicted away from the applied load. A more gradual distribution of pontoon rotation is indicated. Similar to the observation for loading at mid-span, the more gradual distribution may be due to the pontoon structure having a greater torsional stiffness than estimated.

Maximum pontoon rotation for dynamic test cases D4 through D11 is summarized in Tables 7 and 8. The appropriate static test results are included in the table for comparison. Similar to the freeboard loss data presented above, the rotation values shown correspond to the maximum rotation measured at the station immediately adjacent to the location where the trains bypass each other. The time stamp corresponding to the maximum rotation at that station was used to extract data for the other stations. The data is shown graphically in Figures 21, 22, and 27.

Although there are differences in measured rotation between the static and dynamic load tests, no definite trend is evident. On average, the order of magnitude of the difference in bridge response between static and dynamic loading is relatively small compared to the overall

rotation of the bridge. As with freeboard loss, there does not appear to be an amplification or dampening of bridge response due to dynamic loading.

d. West Expansion Joint Movement

Relative movement between both sides of the west transition span expansion joint was measured with string POTs spanning across the joint. String POTs measuring longitudinal movement were installed at the north and south ends of the joint. A third string POT was installed to measure transverse movement at the joint. The string POTs operated properly during the tests and no discrepancies in the data were found.

Maximum measured expansion joint movements for each of the load tests are summarized in Table 9. Horizontal rotation of the joint is also included and was computed from the difference in longitudinal expansion between the north and south sensors. The maximum measured longitudinal movement is 0.45-inches. This correlates well with the predicted maximum longitudinal movement of 0.50-inches. The maximum measured horizontal rotation is 0.02 degrees. This is less than the predicted response of 0.2 degrees per the previous analytical study. A small amount of transverse movement was also measured (0.07-inches).

e. Global Acceleration

Global pontoon acceleration was measured at mid-span and at the west end of the bridge with tri-axial accelerometers installed at instrumentation stations 1 and 5. Data was recorded for acceleration in the vertical direction as well as in the direction of traffic and perpendicular to traffic. During the first night of testing, the accelerometer at station 1 was not working properly. Both accelerometers were functioning during the second night of testing.

Maximum measured accelerations for each of the dynamic load tests are shown in Table 10. A maximum acceleration of 0.05 g's was recorded in the vertical direction at mid-span of the bridge. The maximum acceleration recorded at the west end of the bridge is 0.02 g's vertical.

f. Pontoon Cantilever Tip Deflections

Tilt meters were installed on the underside of the cantilever slab overhang at the south side of the bridge at mid-span to measure the angle of curvature of the slab. Global pontoon rotation was subtracted from the tilt meter readings mounted to the underside of the slab to determine local curvature. Tip deflections of the slab cantilever can be computed from the measured angle of slab curvature. Both tilt meters operated properly during the tests and no discrepancies in the data were found.

Maximum measured tilts at the slab cantilever are summarized in Table 11. The local slab curvature measured by the tilt meters is negligible, indicating that very little local deflection occurred at the tip of the slab due to test vehicle loading.

g. Pontoon Stress

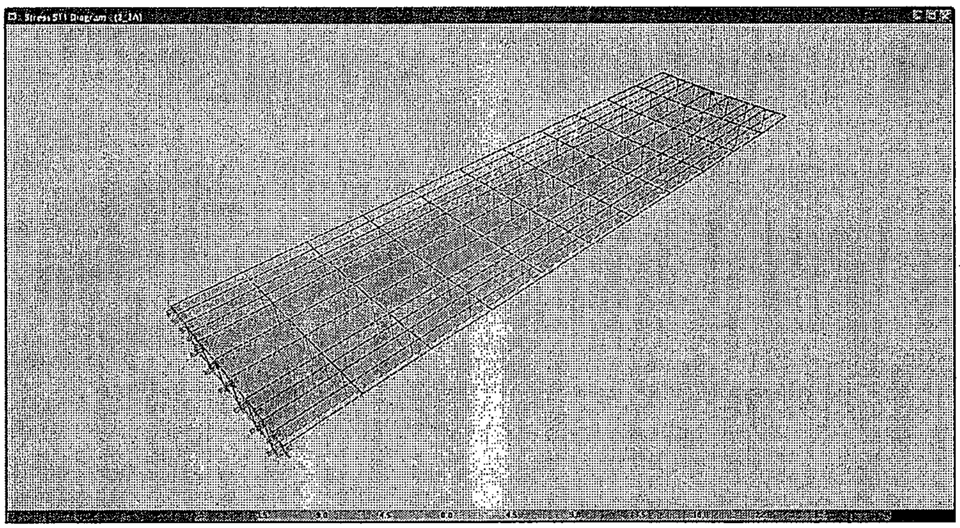
Strain gages were installed at instrumentation stations 1, 2 and 4 to measure global pontoon strain due to test program loading. Four (4) strain gage rosettes and four (4) single gages (for a total of 16 gages) were installed inside the pontoon at each station. The four (4) single gages were oriented in the direction of the longitudinal axis of the bridge. All gages except one operated properly during the tests and no discrepancies in the data were found. The inactive

strain gage was the 45-degree gage that was part of the rosette mounted to the south exterior wall at station 1.

Measured strains were converted to stresses and compared to predicted values. Predicted states of stress at each of the gage locations were obtained per the following procedure:

1. The previously developed 2-dimensional analytical model was used to predict bridge response for each static test case. Values for longitudinal bending moment, torsion, and shear were documented for each instrumentation station for each static test case.

2. A 3-dimensional finite element model (FEM) was created to analyze the stress distribution throughout the pontoon section at each instrumentation station. Loading applied to the FEM corresponded to the moments, torsions, and shears documented in step 1. The FEM is shown below.



Finite Element Model to Analyze Pontoon Stress Distribution

3. Predicted stresses at each of the strain gage locations for each static test case were documented from the FEM. The stresses represent the predicted state of stress due to combined moment, torsion, and shear on the pontoon cross-section.

Comparisons of predicted to measured stress are summarized in Tables 12, 13, and 14. Comparison of maximum measured stress to predicted shows that:

- The maximum measured longitudinal compressive stress is 152 psi, which is 22% greater than the predicted value of 124 psi.
- The maximum measured longitudinal tensile stress is 248 psi, which is 28% greater than the predicted value of 193 psi.
- The maximum measured shear stress is 58 psi, which is 15% less than the predicted value of 68 psi.

Based on the relationship between measured and predicted longitudinal stress, it is recommended that a stress increase factor of 1.30 be applied to longitudinal stresses due to

LRT live load predicted using the analytical method described above. No stress increase factor should be applied to predicted shear stress.

h. Conclusions on Measured vs. Predicted Response

Overall, bridge response correlates well with response predicted using previously developed analytical methods. The following key observations and conclusions are noted:

i. Global bridge deflection and rotation is more distributed along the length of the bridge than predicted, with measured deflection and rotation less than predicted immediately adjacent to the applied load. This may be due to the bridge's actual flexural and torsional stiffness being greater than estimated. The concrete modulus of elasticity value used in the analytical models to estimate structure stiffness is based on the original design criteria for the bridge. Actual concrete strength and stiffness may be greater than that specified in the design criteria. It is feasible that with time the global stiffness of the bridge may decrease as the structure approaches the last third of its service life. A decrease in stiffness would result in LRT live load deflections closer to those predicted than what was measured during the load tests.

ii. There appears to be no definitive trend when comparing global bridge response due to static loading to that for dynamic loading. Globally, the bridge responded similarly for static and dynamic loading.

iii. It is our opinion that continued use of previously developed analytical methods used to study the response of the bridge to LRT live loading is acceptable. It is recommended that a stress increase factor of 1.30 be applied to longitudinal stresses predicted using the analytical methods described above.

5. PONTOON GLOBAL ANALYSIS

Structural feasibility of converting the two HOV lanes of the Homer Hadley Floating Bridge to light rail was investigated in the previous analytical study. In that study, the following conclusions were drawn for the LR (mod) rail system at track location 3:

1. Added weight to the floating bridge due to rail system dead load can be mitigated. Weight mitigation measures were proposed to meet WSDOT's criteria of keeping the bridge in trim and maintaining zero net freeboard loss after the addition of rail system components. Main elements of weight mitigation include:

    a. Removal of existing south concrete barrier and replaced with cable railing.

    b. Removal of existing ballast within the floating bridge pontoon cells.

    c. Removal of 1-inch of the existing concrete overlay on the south side of the concrete median barrier and replacement with 1/4-inch of polymer concrete overlay.

2. The capacity of the top deck of the pontoon is adequate to support LRT provided that the south concrete barrier is removed and replaced with a cable railing.

3. The elevated steel superstructure at the east and west ends of the bridge does not meet Sound Transit's criteria for deflection. A steel cover plate retrofit for the box girders was proposed to mitigate excess deflections. Removal of pontoon ballast directly under the retrofitted box girders is necessary to mitigate added weight due to the steel cover plates.

4. A more refined global analysis is required to evaluate pontoon capacity to support LRT live loading.

The purpose of this study is to address item #4 above by assessing the global capacity of the bridge at mid-span and near the west expansion joint to support LRT live load in combination with other design loads.

a. Study Criteria

The original design criteria documents for Stage 1 and Stage 2 construction were used as the basis for this study, copies of which are included in Appendix F. The design criteria provides information on the types of design loads and load combinations to be used in the design of the bridge, analysis methods and codes, allowable stress criteria and performance criteria. LRT live loading is included in the document and is considered as part of the overall live load on the bridge, in combination with traffic live loading.

Global moment and torsion demand on the bridge was determined using the design service-level load combinations that include live load. The applicable load combinations are shown below:

Group	Loads	% of Basic Allowable Stress
1	D + H + L + I + K	100
3	Gr. 1 + WN + NW + WL + LF	125
4	Gr. 1 + T + S	125
6	Gr. 3 + T + S	140
8	Gr. 1 + DM	140
10	Gr. 3 + DM	150

Where:

- D = Dead load.

  As directed by WSDOT, dead load effects on global demand were ignored under the assumption that pontoon ballast completely trims all dead load effects.

- H = Hydrostatic Pressure.

  Hydrostatic pressure demand on the bottom slab and exterior walls of the pontoons was obtained from WSDOT's original design calculations for the bridge.

- L = Live Load.

"Live Load" represents the maximum response due to the combination of LRT and HS25 traffic loads (HS20 traffic loads are indicated in the design criteria document). HS25 traffic loading on the westbound roadway was added to the 2-dimensional computer model of the bridge in various patterns to accentuate response due to LRT live load at mid-span and at the west end of the bridge. Influence lines were used to determine placement of the HS25 loads to maximize response. An envelope of response was output for LRT live load in combination with traffic loading to produce maximum values for moment and torsion. Reduction factors for multiple lane loading are included in Live Load, with each track of LRT considered as a lane of loading. The reduction factors used are as follows:

- One or two lanes loaded = 1.0
- Three lanes = 0.90
- Four or five lanes = 0.75
- Six lanes = 0.60

Maximum global stresses within the pontoon cross-section due to live load moments and torsions were evaluated by using a 3-dimensional finite element model of the pontoon cross-section. Longitudinal stresses due to LRT live load were increased using a scale factor of 1.30 to account for the difference between measured and predicted stress.

- I = Live Load Impact.

  Per the design criteria, live load impact is not to be included for evaluation of global pontoon stresses.

- K = Change in lake level

  Not included in global analysis, however changes in lake level will change the geometry of the transition spans, resulting in expansion joint movement.

- WN/NW = Normal wind/wave on structure – 1-Year Storm

  Bridge moment and torsion response due to wind/wave loading for the 1-year storm is provided in the 1983 hydrodynamic study performed by the Glosten Associates titled, "Wave Loading Analysis of Lake Washington Bridges, and Results, New I-90 Floating Bridge". The applied moment resulting from the load corresponding to the breaking strength of one transverse anchor cable is also included in this load case. This load represents the maximum cable reaction applied to the bridge just prior to breaking during the 1-year storm. A maximum breaking strength of 300 tons is assumed for the cable.

- WL = Wind on live load.

  Included in Glosten's hydrodynamic study.

- LF = Longitudinal force due to live load

- T = Temperature

  Moments due to temperature effects per the original design calculations for the bridge were provided by WSDOT.

- S = Shrinkage & Creep

- DM = Damage.

  Moments due to damage corresponding to flooding of all cells across the width of the pontoon adjacent to a transverse anchor wall per the original design calculations for the bridge, provided by WSDOT.

Pontoon prestress forces are also included in all of the above load combinations and were obtained from WSDOT's original design calculations for the bridge.

Service-level demand stresses resulting from the above load combinations were compared to allowable stresses consistent with the design criteria for the bridge.

For non-prestressed members, the following basic allowable stresses were used:

- Allowable rebar stress in sections resisting sustained hydrostatic forces = 14,000 psi
- Allowable rebar stress in sections not resisting sustained hydrostatic forces = 24,000 psi

For prestressed members, the following basic allowable stresses were used:

- Allowable concrete compression = 0.6*f'c = 2,400 psi.
- Allowable tension in pre-compressed tensile zone:
  - Service group loadings not containing the damaged condition:
    - 0 psi
  - Service group loadings containing the damaged condition:
    - $3\sqrt{f_c'}$ in transverse direction, with mild reinforcement carrying the tensile stress. Effect of "percent of basic allowable stress" is not allowed.
    - 0 psi in longitudinal direction, however, concrete tensile stresses up to $3\sqrt{f_c'}$ may be resisted by mild reinforcement when considering "local" bending stresses.
- Allowable rebar stress in sections resisting sustained hydrostatic forces = 14,000 psi
- Allowable rebar stress in sections not resisting sustained hydrostatic forces = 24,000 psi

Demand stresses were compared to allowable by reducing demand by the appropriate reduction factor corresponding the allowable stress increase for each load combination, as shown below for Group 3 loading:

$$\left[\left(\frac{D_{Group3}}{1.25}\right) + prestress\right] \leq or \geq Basic\ Allowable\ Stress$$

Per the design criteria, the ultimate flexural strength of the global pontoon section shall not be less than:

1.3 (D + H +K) + 2.17L

The ultimate capacities of the pontoons for both moment and torsion are provided in the Glosten Associates December 1994 report.

b. Study Results

Mid-span

The bottom slab, exterior walls, and interior longitudinal walls of the mid-span pontoon are prestressed in the longitudinal direction only. The top slab of the pontoon is prestressed in the transverse direction only. Results for the critical service-level stress conditions at mid-span are summarized below. Controlling load combination groups are indicated with the demand values. Additional information is contained in Tables 15 through 20.

Bottom Slab - Longitudinal (prestressed) direction:

Location	Condition	Stress Demand (psi)	Stress Allowed (psi)	
Bottom Slab	Max. concrete tensile stress	-32 (compression, Gr. 6)	0	O.K.
	Max. concrete compressive Stress	1,242 (Gr. 6)	2,400	O.K.

Bottom Slab – Transverse (non-prestressed) direction:

Location	Condition	Stress Demand (psi)	Stress Allowed (psi)	
Bottom Slab	Max. rebar stress	13,400 (Gr. 1)	14,000	O.K.

South Exterior Wall – Vertical (non-prestressed) direction:

Location	Condition	Stress Demand (psi)	Stress Allowed (psi)	
Exterior Wall	Max. rebar stress	7,050 (Gr. 1)	14,000	O.K.

Top Slab – Longitudinal (non-prestressed) direction:

Location	Condition	Stress Demand (psi)	Stress Allowed (psi)	
Top Slab	Max. rebar stress	25,170 (Gr. 6)	33,600	O.K.

The maximum service-level principal tensile stress in the non-prestressed direction caused by combined torsion, moment and prestress on the pontoon section is shown below:

Maximum Principal Stress (Tension, psi)	Stress Allowed (psi)	
182 psi (Gr. 6)	212	O.K.

Comparison of factored global moment to ultimate global moment capacity is shown below:

	Factored Moment, ft-k	Ultimate Capacity, ft-k	D/C
Max. Positive Moment	101,625 (Gr. 1)	208,000	0.49
Max. Negative Moment	–38,759 (Gr. 1)	-127,000	0.31

Comparison of service-level torsion for Live Load plus the 1-year storm event to ultimate torsion capacity is shown below:

Case	Service-level Torsion Demand, ft-k	Ultimate Capacity ft-k	D/C
Live Load	34,298	45,500	0.75
Live Load + 1-Year Storm	44,298	45,500	0.97

The above results indicate that strength and serviceability criteria are met for LRT live load applied in combination with other design loads at the mid-span of the bridge.

West End of Bridge

Pontoon B at the west end of the bridge is prestressed in the longitudinal, transverse and vertical directions. Results for critical service-level stress conditions at Pontoon B are summarized below. Additional information is contained in Tables 15 through 20.

Allowable Stress Evaluation at Pontoon B for Longitudinal Stresses:

Location	Condition	Stress Demand (psi)	Stress Allowed (psi)	
Bottom Slab	Max. concrete tensile stress	–314 (compression, Gr. 10)	0	O.K.
	Max. concrete compressive stress	1,523 (Gr. 6)	2,400	O.K.
Top Slab	Max. concrete tensile stress	–907 (compression, Gr. 4)	0	O.K.
	Max. concrete compressive Stress	2,148 (Gr. 10)	2,400	O.K.

The maximum service-level principal tensile stress in the non-prestressed direction caused by combined torsion, moment and prestress on the pontoon section is shown below:

Maximum Principal Stress (Tension, psi)	Stress Allowed (psi)	
34 (Gr. 10)	212	O.K.

Comparison of factored global moment to ultimate global moment capacity is shown below:

	Factored Moment, ft-k	Ultimate Capacity, ft-k	D/C
Max. Positive Moment	116,451 (Gr. 1)	627,000	0.19
Max. Negative Moment	-40,325 (Gr. 1)	-616,000	0.07

Comparison of service-level torsion for Live Load plus the 1-year storm event to ultimate torsion capacity is shown below:

Case	Service-level Torsion Demand, ft-k	Ultimate Capacity ft-k	D/C
Live Load	23,218	81,300	0.29
Live Load + 1-Year Storm	35,218	81,300	0.43

The above results indicate that strength and serviceability criteria are met for LRT live load applied in combination with other design loads at the west end of the bridge.

c. Conclusions

The results of this study indicate that WSDOT's strength and serviceability criteria for pontoon global response are met at mid-span and at the far west end of the bridge for live loading due to two (2) tracks of Sound Transit's LRT system in combination with HS25 traffic on the westbound roadway. A comprehensive analysis of all remaining pontoons of the bridge should be done during final design. Also, as concluded in the previous analytical study, response of the bridge to LRT live loading must be evaluated by Sound Transit's rail designers for conformance to light rail system tolerances.

6. MOTION ANALYSIS AT WEST TRANSITION SPAN

Motion analyses were performed for the west transition span due to combined LRT live load, HS25 traffic load, and 1-year storm loading. Motions due to LRT and traffic loads were obtained from the 2-dimensional computer model used for the global analysis of the floating bridge. Motions due to storm loading were obtained from the 1983 hydrodynamic study performed by the Glosten Associates. All motions were translated to the elevation at the top of roadway. A summary of the results is shown below. Additional information is contained in Table 21 and Figures 29 & 29.1 .

Transition Span Rotation:

Horizontal rotation of transition span due to transverse displacement of floating bridge, θ_H	0.12 degrees
Vertical rotation of transition span due to vertical displacement of floating bridge, θ_V	0.26 degrees

Movement at joint between fixed approach structure and transition span at Pier 7:

Max. longitudinal expansion/contraction of joint, Δ_{x7}	±0.8 inches
Max. transverse displacement of deck	0 inches
Max. vertical displacement of deck	0 inches

Movement at joint between transition span and floating bridge at Pier A-1:

Max. longitudinal expansion/contraction of joint, $\Delta_{x\,A\text{-}1}$	±1.2 inches
Max. transverse displacement of deck, Δ_y	4.7 inches
Max. vertical displacement of deck, Δ_Z	10.0 inches

Transition span motion due to changes in lake level is not included above. The design criteria for the bridge indicates a maximum rise of 0.8-feet and a maximum fall of 3.8-feet from the normal water elevation of 8.02-feet. Sound Transit's rail designers should include this component of motion in the design of the rail system.